Geotechnical Design Considerations for Contained Aquatic Disposal

PURPOSE: This technical note provides geotechnical design guidance for contained aquatic disposal (CAD) sites. The geotechnical behavior of the recently deposited dredged material with respect to consolidation, bearing capacity, and stability are major factors that are considered. The presence or absence of a cap may be considered in each analysis procedure.

BACKGROUND: As disposal of contaminated sediments becomes more constrained (e.g., regulated and costly), U.S. Army Corps of Engineers District offices look to various and innovative disposal options for this material. One of the more promising is CAD. This form of dredged material disposal involves controlled placement of dredged material into a subaqueous site with some form of lateral confinement. The lateral confinement may be provided by a bottom depression or by subaqueous berms. The contaminated material is then capped in most instances with clean sediment to physically separate it from the overlying environment.

CAD is usually used in situations where the properties of the contaminated material and/or bottom conditions (e.g., slopes) require positive lateral control measures during placement. Use of CAD can also reduce the required quantity of capping material; thus the costs for disposal may be lowered. Options might include the use of an existing natural depression, pre-excavation of a disposal pit, or construction of one or more submerged dikes for confinement (Palermo et al. 1998b).

The majority of CAD projects use excavated borrow pits or cells to contain the contaminated dredged material. The excavated depth of these cells ranges from a few feet to 9-15 m (30-50 ft), or more. The horizontal dimensions of the cells vary considerably, but some of the larger cells have approached 457 m (1,500 ft) wide by 1,524 m (5,000 ft) long. The cells at many of the larger sites are filled segmentally; that is a smaller portion (an end or a corner) is filled with contaminated sediments and capped. Other sections of the cell are used later for CAD development. These initial deposits in the CAD may result in formation of mounds or sloped deposits that may overlap each other or cover the entire bottom of the CAD cell. With such a sequential plan of filling combined with regulatory requirements for placing and maintaining a viable cap of given thickness on top of the contaminated material, it is essential that geotechnical issues be addressed in the design of these CAD projects. In other situations, very soft, sometimes hydraulically dredged and placed material is to be capped. In these instances, the ability of the dredged material to sustain a viable cap on its surface, i.e., bearing capacity, must be evaluated. On some recent CAD projects, problems have been encountered with keeping the cap above the very soft contaminated sediments.

Although many of the chemical, biological, hydraulic, and operational requirements associated with CAD sites have been fairly well defined, much of the geotechnical behavior has been ignored. However, several recent CAD projects have focused attention on geotechnical issues. The 1997 closure of the U.S. Army Engineer District, New York, Mud Dump Site, with its associated maximized final use coupled with intense monitoring efforts and an expedited capping schedule,

highlighted potential problems with slope stability and bearing capacity of the soft dredged material (Clausner et al. 1998; Rollings and Rollings 1998a; Rollings and Rollings 1998b). The 1998 Boston Harbor CAD demonstration project highlighted potential problems with sustaining a viable cap on the surface of the recently deposited soft dredged material (Murray et al. 1998; ENSR 1997). Other Districts have used or are considering using CAD as a disposal option (e.g., Los Angeles and Seattle). With the popularity of and interest in CAD, a need was recognized for geotechnical design guidance.

Evaluation and understanding of consolidation and shear strength behavior of these soft deposits will allow assessment of long-term site capacity and stability/physical isolation capabilities of the cap, respectively. In CAD deposits, slope stability is of concern when the CAD cell is to be partially filled across its aereal extent; it is also of potential concern regarding the side slopes of the CAD cell. Bearing capacity, or the ability of the contaminated dredged material to support the capping material, must be determined if the contaminated material is to be successfully isolated from the overlying environment. It is essential that geotechnical design and analysis be conducted for these CAD projects so that they function as sound engineering structures with behavior that can be quantified and accurately predicted.

NEED FOR GEOTECHNICAL EVALUATIONS: Geotechnical engineering considerations are important in design and construction of CAD sites, particularly since most contaminated sediments are fine-grained silts and/or clays and have high water contents and low shear strengths in situ, a most daunting combination of factors. As these sediments are dredged and placed at a subaqueous containment site, the situation is often exacerbated by the mixing or inclusion of more water in the sediments, resulting in even lower shear strengths. For normal geotechnical construction projects, typical requirements are high shear strength and low moisture content, just the opposite of conditions existing in recently deposited dredged material.

Soil shear strength is a complex function of many parameters including soil density, particle distribution, moisture content, and plasticity characteristics. Assessment of the shear strength is important because it controls (a) the bearing capacity of the dredged material, which then determines the ability of the dredged material to hold up (sustain) a cap of clean sediments, and (b) the stability of any slopes of dredged material that may form in the CAD, e.g., the stability of a mound of dredged material in the bottom of the CAD when only a portion of the footprint of the CAD is covered with material. Over time, as the dredged material undergoes consolidation, the shear strength of the sediment will increase somewhat and its ability to maintain a viable sand cap of given thickness will likely improve. The degree of change of the shear strength will depend upon material type, the initial conditions of the dredged material, the thickness of the deposit, and the thickness of any overlying cap that will act as a surcharge load. The most critical time with regard to slope stability is immediately after disposal and/or immediately after capping when the pore pressures will be greatest and excess pore-water pressure has not had time to dissipate. Likewise the most critical time regarding bearing capacity is immediately upon placement of the cap while the shear strength of the contaminated sediments is at its lowest.

Because contaminated sediments are usually fine-grained and have a relatively high water content, they are often susceptible to large amounts of consolidation, a phenomenon that entails the squeezing/pressing together of soil particles as pore water is expelled under sustained load. Assessing consolidation potential of capped dredged material deposits requires consideration of the

consolidation potential of three elements: the cap, the contaminated dredged material, and the native or substrate sediments. Quantifying consolidation is necessary to meet environmental, regulatory, and economic requirements. Quantification of consolidation addresses three issues.

- First, changes in elevation due to consolidation must be differentiated from those due to erosion. Decreases in elevation of the surface of the deposit caused by erosion will likely require remedial actions to replenish and restore the cap to its required thickness.
- Second, consolidation should be considered when determining long-term site capacity. As the CAD deposit consolidates and decreases in elevation, volume within the facility becomes available for storage of additional dredged material.
- Third, a consolidation analysis will provide data needed to evaluate the potential movement of pore water from the contaminated sediment upward into the cap, a necessary analysis in evaluating the potential for long-term flux of contaminants.

While *hydraulic dredging* may result in a CAD deposit with sand or clay ball deposits immediately below the discharge pipeline and thus segregated from the majority of the fill, the remainder of the dredged material is normally relatively homogeneous as a result of mixing during dredging and disposal. The undefined and unknown macrotexture of *mechanically dredged* and subaqueously placed dredged material deposits makes analysis of these deposits extremely difficult. When some materials are mechanically dredged, almost all of the *clump* or *bite* of dredged material contained within the clamshell bucket may retain its in situ consistency. In other cases (at higher water contents), the material from each clamshell bucket may flow together to form a relatively homogeneous amalgamation of material with a minimum of seams, pockets, or veins of wetter material. The geometry of not only the deposit, but also the homogeneous material and the location and extent of heterogeneous pockets and seams, directly affects the behavior of a soil deposit. These factors are critical for an accurate engineering assessment of soil behavior. For consolidation analyses, this does not pose too difficult a problem, as the process of consolidation considers the gross behavior of the mass of the deposit. The only discontinuities or rapid material changes that would have a significant effect on consolidation would be continuous, free-draining layers (such as continuous sand layers) which are highly unlikely to occur in typical dredged material deposits. Even if such layers were present, they would only shorten the time required for consolidation; they would not change the ultimate consolidation that would occur.

However, matters of shear strength present an entirely different situation. When dealing with shear strength of geotechnical materials and shear failures in deposits of these materials, it is imperative to remember that the average strength means nothing regarding deposit stability. While average values can usually be used for consolidation (unless structures that cannot undergo differential settlement are going to be built) and for many other physical, chemical, or biological properties or conditions, average values are completely useless for assessing the strength condition and stability of deposits. This is because the stability of a soil deposit is controlled by the weakest layer or stratum in or under the deposit of interest. Thus one very thin weak seam will become the potential failure plane, and the fate of the deposit depends upon the strength of this one layer. For subaqueous dredged material deposits, particularly mechanically dredged material, conditions within the deposit are not and cannot be determined to the level normally desired/required to conduct an exacting slope

stability analysis. Thus the accuracy of the analyses will be limited until actual conditions within CAD deposits can be more accurately determined.

Only very limited geotechnical evaluations have been considered in past CAD and other capping projects. In virtually all of the past projects, the design was empirical, i.e., prior field experience showed that it worked, but actual geotechnical design calculations were not made. Development and application of appropriate geotechnical design guidance should result in better designs.

MATERIAL PROPERTY DETERMINATION: Before any geotechnical analyses and evaluations can be conducted, the physical properties that control engineering behavior must be determined for the soil mass. To do this requires that representative samples of the dredged material be collected and tested and the results be used in geotechnical analyses to predict engineering behavior.

Sample Collection. Sample collection must be conducted in a manner that will ensure that the material tested is representative of the dredged material that has been or will be placed in the CAD cell/site. Since sediments will be disturbed (at least to some extent) during the dredging process, it will not be necessary to obtain truly undisturbed samples for laboratory classification and consolidation testing; however, for shear strength determination, undisturbed samples are required. If sampling is to be completed prior to dredging, use of a coring device that can penetrate the sediment may be required if the depth, consistency, or variability of the deposit prevents adequate, representative sample collection with a grab sampler. If the depth to be dredged is small and the sediments have been recently deposited and have been frequently mixed by ship traffic, it will be sufficient to collect grab samples using some kind of sampler or mechanical dredge, e.g., Petersen dredge. If sampling is to be accomplished during dredging, samples may be taken periodically in 0.02-cu-m (5-gal) buckets (or other suitable containers) throughout the dredging process. If mechanical dredging is used, samples for laboratory testing can simply be recovered from the dredged material as it is placed in the barge for transport. More specific guidance regarding sample collection and preservation is given in the *Green Book* (U.S. Enviromental Protection Agency (USEPA)/U.S. Army Corps of Enginers (USACE) 1991).

For classification and consolidation testing, the collected samples may be combined to form one homogeneous composite sample unless there is additional information regarding material type, placement operations, etc., that would preclude compositing. The idea here is to have a laboratory sample that is the most representative possible of the sediment in the CAD deposit. Therefore, it is necessary to have some decisions about material representative of the sediment made by individuals familiar with the specific project.

Material Characterization Testing. After samples are collected and composited, they must be tested by a competent geotechnical testing laboratory. The following tests should be performed to properly characterize the material as a first step in predicting its engineering behavior:

- Natural water content, solids concentration, and/or density.
- Grain size distribution.
- Plasticity indices, i.e., Atterberg limits.

- Unified Soil Classification System classification.
- Specific gravity.
- Organic content.

Standard geotechnical laboratory test procedures, such as those of the American Society for Testing and Materials (ASTM), should be used for each test. Table 1 gives the standard ASTM designation for the needed tests; it also cross-references the ASTM procedures to those of several other organizations that have standardized test methods. As discussed in the following paragraphs, nonstandard laboratory testing must usually be used for shear strength, consolidation, and permeability determinations in these very soft dredged materials.

Table 1
Standard Laboratory Test Procedures

	Test Designation				
Soil	**ASTM**	**AASHTO[1]**	**COE[2]**	**DoD[3,4]**	**Comments**
Water content	D 2216	T265	I	Method 105, 2-VII	
Grain size	D 422	T88	V	2-III, 2-V, 2-VI	
Atterberg limits	D 4318	T89 T90	III	Method 103, 2-VIII	
Classification	D 2487		III		
Specific gravity	D 854	T100	IV	2-IV	
Organic content	D 2974				Use Method C
Consolidation[5]	D 2435	T216	VIII		
Permeability[6]	D 2434	T215	VII		
Shear tests	D 2573 D 4648				Field test Laboratory test

[1] American Association of State Highway and Transportation Officials.
[2] Headquarters, USACE (1986).
[3] Department of Defense (1964).
[4] Department of the Army (1987).
[5] Do not use the ASTM laboratory test for determining consolidation. Instead, use the modified standard consolidation test and the self-weight test (Headquarters, USACE, 1987; Cargill 1983; Poindexter 1987, 1988).
[6] One value of pemeability must be calculated from the self-weight consolidation test.

Shear Strength Testing. Determination of the shear strength of very soft soils is extremely difficult. Because of the soft conditions of the materials, they are difficult to sample, transport, and test in an undisturbed condition. Many of these soils are too soft to stand under their own weight; thus conventional triaxial testing, unconfined compression testing, and direct shear testing are impossible or impractical to run. The most commonly used laboratory testing technique for very soft sediments is the laboratory vane shear device. Although it does not always correlate directly to field vane testing, it is much more economical and is used extensively in the marine sediment industry. Because of its extensive use and the large volume of strength data it has generated, the laboratory vane shear test is well accepted in the marine geotechnical engineering community.

Ideally, the vane shear test would be conducted in the field to prevent any change in strength that might be associated with any sampling procedure used in this soft material; however, if relatively undisturbed core samples are collected, laboratory vane shear tests can be run to provide an indication of shear strength. Laboratory tests are often more feasible because of logistics and cost considerations. ASTM D 2573 (ASTM 1999a) and D 4648 (ASTM 1999b) give the procedures for field and laboratory vane shear tests, respectively.

A vane strength is determined on the *undisturbed* material. The test is performed by inserting a vane of known dimensions into the sample so that the top of the vane is at least one vane length deep in the soil and rotating the vane at the prescribed rate of rotation; the rotation rate is then correlated to soil strength. A remolded strength is subsequently determined on each sample. The remolded strength is obtained by performing the original test to failure, and then rotating the blade through five revolutions and re-performing the test at that same testing location. By so doing, the material tested is the remolded sediment at the edges of the vane.

For a recent project for New York District, a laboratory vane shear test device was used to obtain shear strength values for the materials collected at the Mud Dump, New York District's former offshore disposal site. Two vanes were used in the testing device for this project; both vanes had 12.7-mm- (0.5-in.-) diam blades but the lengths of the blades were different (12.7 and 25.4 mm (0.5 and 1 in.)). The size of the blade used was determined by the consistency of the material to be tested; the larger blade was used for softer materials. The rate of shearing was set in accordance with standard specifications. To run the vane shear test, sections from the cores were selected for testing at specified locations. Sections 76.2 to 101.6 mm (3 to 4 in.) long were cut from the cores, and the material was tested while in the coring tube. By testing the material in the tube, additional disturbance of the material was minimized and potential problems with the material not being able to stand under its own weight were eliminated.

Consolidation Testing. Laboratory consolidation testing of soft soils often requires use of two types of consolidation tests. Both a modified version of the standard oedometer (consolidation) test and a self-weight test must often be conducted; these tests provide data for the low and high ends of the anticipated range of void ratios, respectively. However, on relatively firm dredged materials that are mechanically dredged, use of oedometer testing alone may suffice.

For consolidation testing of soft materials, a modification to the ASTM D 2435 (standard oedometer test) (ASTM 1999c) loading sequence is required, as outlined in Appendix D of Engineer Manual (EM) 1110-2-5027 (Headquarters, USACE, 1987). A lighter loading sequence is necessary on soft sediments because the material cannot sustain the normal set of loads; i.e., the bearing capacity of the dredged material is too low. Maximum void ratios that can be tested in the oedometer are 5 to 6.

If the void ratios in the field deposit will exceed those that can be simulated in the oedometer test, then a self-weight test will be required to provide compressibility data at the higher void ratios. This test allows a slurry of dredged material to undergo self-weight consolidation in the 152.4-mm- (6-in.-) diam, 304.8-mm- (12-in.-) high consolidometer. Deformation measurements are taken over time, and the device is then disassembled for incremental sampling of the specimen. Typical void ratios encountered in the specimen after completion of consolidation range from 5 to 12 (from

bottom to top of the specimen). This test, developed by Cargill (1983), has been described in detail by Poindexter (1987, 1988).

Samples of site water should be collected for use in mixing and diluting the sediments (for self-weight consolidation testing, if needed). Because the salt content of a sediment and its pore water can significantly affect the engineering properties of the material, it is necessary to use the proper concentration of salt in the pore water to accurately simulate the field behavior of the material; this is most easily done by using water from the dredging/disposal site. If there is a significant difference between salinity at the dredging and disposal sites, it is recommended at this time that disposal site water be used if the sediment will drop freely through the water column upon disposal.

If compressible materials are encountered below the CAD cell, these materials should be tested according to ASTM D 2435 (standard oedometer test) (ASTM 1999c) to determine their compressibility. If compressible capping material will be used at the CAD, this material should be tested in the oedometer, possibly supplemented by the self-weight test if field void ratios indicate it is needed.

Permeability Determination. Permeability of fine-grained soils is usually determined by a falling head permeability test, which is run in conjunction with the (oedometer) consolidation test. The procedure for this test is given in ASTM D 2435 (ASTM 1999c) and EM 1110-2-1906, Appendix VII, Part 8 (Headquarters, USACE, 1986). When a permeability test is run on dredged material, care must be taken with these very soft materials to prevent consolidation caused by the force of water seeping through the sample (known as seepage consolidation).

As an alternative to running a permeability test, the permeability may be calculated from consolidation properties by using the following relationship (Lambe 1951; Terzaghi, Peck, and Mesri 1996).

$$k = \frac{T_{50} H^2 \gamma_w \alpha_v}{(1+e)t} \tag{1}$$

where

k = permeability, ft/sec

T_{50} = time factor = 0.197

H = average height of soil sample for load increment, ft

γ_w = unit weight of water, lb/ft^3

α_v = average coefficient of compressibility = $-\Delta e / \Delta \sigma'$

e = average void ratio for load increment

t = time, sec

The value of α_v is computed from the consolidation test data. This equation is used to calculate permeability of the soil specimen in the consolidation test for each load increment. Average values

of height H, void ratio e, and coefficient of compressibility α_v over each load increment are used in the calculations. More detailed information on use of this equation is contained in Lambe (1951) and Terzaghi, Peck, and Mesri (1996).

GEOTECHNICAL ENGINEERING ANALYSIS: The elevation changes in a CAD deposit could conceptually arise from consolidation of the dredged material as soil particles are rearranged to a more dense state accompanied by an expulsion of pore water or from shear displacements within the material. Consolidation of dredged materials is recognized as an important phenomenon affecting site capacity (Poindexter 1988; Rollings 1994; Rollings and Rollings 1998a); however, it is a long-term issue. Conversely, shear displacements are the more likely cause of large, rapidly occurring elevation changes, particularly soon after placement of the dredged material and/or capping material. These shear displacements may develop from collapse, bearing capacity failure, or slope failures within the dredged material.

The process of consolidation occurs in fine-grained soils as soil particles are pressed together under load. Consolidation may occur in the capping material (if a compressible material is used for the cap), in the contaminated sediments, and/or in the underlying foundations soils. In the capping material, consolidation will result from self-weight of the material, while in the underlying contaminated sediment, consolidation will occur firstly from self-weight and secondly as the result of the load imposed on it by the capping material. Consolidation of the natural bottom underlying the recently constructed CAD will occur only if the load caused by the contaminated sediments and cap exceeds any load previously placed on the foundation soils. For example, if the weight of the material excavated during construction of a depression for a CAD exceeds the weight of the dredged material and cap, then little or no foundation consolidation would be expected.

It should be noted that some rebound may occur upon unloading of the underlying soil (by excavating the cell). Rebound would be indicated by an upward movement of the bottom surface of the CAD cell. In most instances, this effect would likely be negligible. However, in some cases, it might affect pay yardage for excavation of the cell as well as the holding capacity of the CAD cell. The magnitude of any possible rebound can be determined during the consolidation test by unloading the specimen after each consolidation load increment. Procedures for determining rebound are given in the consolidation test procedures (ASTM 1999c; Headquarters, USACE, 1986).

Consolidation will generally occur in most fine-grained soils, although the amount and rate can vary greatly depending upon a number of factors including the particle type (e.g., clay versus silt, high versus low plasticity), moisture content/density, and permeability of the deposit, combined with the loading conditions and thickness of the compressible layers. All of these factors interact to affect the compressibility of sediment layers significantly. Consolidation is a slow process, sometimes taking years or decades to reach completion. Coarse-grained sediments, e.g., sands and gravels, will not consolidate appreciably.

Collapsible soils are formed in nature by various geologic processes in a variety of deposition environments and can also be found in engineered earthworks (Mitchell 1993; Rollings and Rollings 1996). In simple terms, the particles of collapsible soils are deposited in approximately point-to-point contact in a meta-stable condition. Upon loading or after dissolution of any bonding agent,

the particles reorient or collapse into a denser state. The clumps of dredged material in a matrix of soft slurry from clamshell dredging could conceptually be in such a meta-stable condition, but too little is known of the actual deposition conditions of a dredged material deposit for this to be any more than a hypothesis. Even if such a collapsible structure were built into these man-made deposits, knowledge of the collapse behavior or quantitative methods of analyzing the magnitude of the potential collapse does not exist.

Both bearing capacity and slope failures are conventional geotechnical limit equilibrium problems and are routinely analyzed in practice. In such problems, the factor of safety against failure is calculated by dividing the available resistance due to the soil shear strength by the sum of the shear stresses along a predefined slip or failure plane. These shear forces may develop from surface loading, as is typically encountered in foundation problems, or they may be caused by gravity loads reflecting the specific geometry of a slope as well as any superimposed surface loads. At a factor of safety of 1.0, the stresses just equal the available soil strength, and the loading condition is nominally stable along the analyzed plane for any factor of safety of 1.0 or greater. In practice, bearing capacity factors of safety are typically 3 to 5, and slope stability factors of safety are often 1.3 or higher. Specific magnitudes of factors of safety used in design reflect the uncertainty concerning soil or loading conditions or the seriousness of the failure occurring. Bearing capacity failures in a soil mass develop when applied surface loads exceed the available soil strength. Failure is a function of the magnitude of the load, size of the loaded area, and soil strength. Analysis of slope stability usually requires analyzing a variety of trial surfaces and identifying the failure surface with the lowest factor of safety. The results of slope stability calculations are heavily dependent on the available soil strength, density of the soil, location of phreatic surface, and slope geometry (e.g., height, length, and angle of slope).

As stated earlier, the most critical time for shear displacements is relatively soon after deposition. At this time, excess pore water pressures have not drained, so the analyses should use undrained conditions. This is also called the $\phi = 0$ condition. For slope stability problems, the most critical geometries or worst-case conditions are steep slopes combined with thick dredged material deposits. It is very obvious that the actual slopes attained in most dredged material deposits are quite small when drawn to scale because of the low shear strength of the material; in fact, many mounds of dredged material resemble pancakes on a griddle.

Other more unusual behaviors have been postulated concerning the behavior of soft dredged material below a sand cap. These include:

- A relatively rigid sand cap floating on fluid-like dredged material, then tipping to allow the dredged material with excess pore pressure to escape to the surface of the cap.
- A relatively rigid sand cap through which the pressurized dredged material erupts much as a volcano erupts from the surface of the earth.
- Multiple localized bearing capacity failures that allow columns of sand to penetrate the dredged material until an equilibrium condition is reached in which the sand bridges the area between the sand columns (this would be somewhat similar to the concept of stone columns used to provide foundation support over very soft ground). The validity of such unusual behavior cannot be assessed without further laboratory and field investigations. Whatever

the mechanisms involved, the problem of soft dredged material underlying a firmer cap is a gravity problem with several potential failure mechanisms, and the controlling mechanism cannot be identified until more intense investigations are conducted.

Bearing Capacity Analysis. The bearing capacity of a soil subjected to an applied load of limited extent is a classical geotechnical engineering problem in plastic equilibrium analysis that is covered in almost any text on soil mechanics or foundations (e.g., Terzaghi, Peck, and Mesri 1996; Peck, Hanson, and Thornburn 1974; Sowers and Sowers 1970; or Teng 1962). For this analysis, a portion of the sand cap is modeled as a long surface load that is narrow with respect to the total width of the dredged material deposit, and any shallow slopes are neglected. This would represent a condition where the sand cap is partially in place over a portion of the dredged material, and mathematically corresponds to classical solutions for plane strain conditions used for analysis of continuous footings. The bearing capacity of the dredged material is calculated using the equation for a continuous footing from Teng (1962)

$$q_{ult} = \frac{Q}{A} = cN_c + \gamma D N_q + 0.5\gamma B N_\gamma \tag{2}$$

where

q_{ult} = ultimate bearing capacity, lbf/ft^2

Q = ultimate bearing capacity, lb

A = area of footing, ft^2

c = cohesion (strength) of soil, lbf/ft^2

N_c, N_q, N_γ = Terzaghi's bearing capacity factors

γ = unit weight of soil (submerged weight for the dredged material deposit), lb/ft^3

D = depth of foundation measured from low side of the ground surface to bottom of footing, ft ($D = 0$ for this case)

B = width of footing, ft

For undrained conditions ($\phi = 0$), N_γ is zero, and the third term in the equation goes to zero. For a surface load such as a cap, D is 0, so the second term also goes to 0. Standard texts usually provide graphical solutions for the Terzaghi bearing capacity factors, which give somewhat different answers depending on the scale used in the figure. For the specific case of plane strain conditions under a surface load as in this case, N_c can be determined to be 5.14 (Prandtl 1921). The bearing capacity equation then is reduced to

$$q_{ult} = 5.14c \tag{3}$$

Some authors have suggested reducing the value of cohesion in this equation by one third, i.e., using $2/3c$ (Palermo et al. 1998a; Ling et al. 1996). This apparently comes from interpretation of a rule of thumb stated in Terzaghi and Peck (1967) that was intended to reduce the surface deflections in

structures built on soft soils. However, there is no theoretical basis for this reduction, and the newer version of Terzaghi, Peck, and Mesri (1996) does not include any reference to a reduced value. Since any reduction in the soil strength value will result in a reduced bearing capacity and thus a thinner cap that can be sustained on the surface of the deposit, it is advantageous to use the full value of cohesion when evaluating feasibility of capping. Therefore, it is recommended that the value of cohesion (strength) used in Equation 3 not be reduced unless and until data have been collected to indicate a need for the reduction. It should also be noted that recent analyses of dredged material deposits in New York District agreed well with field performance, and no reduction in the strength value was assumed in these analyses (Rollings and Rollings 1998b).

Using the density of the sand cap, the factor of safety (FOS) against a bearing capacity failure of the sand cap on the dredged material can then be calculated as

$$FOS = \frac{Available\ Bearing\ Capacity}{Load\ Applied\ by\ Sand} = \frac{q_{ult}}{h\gamma_s} \tag{4}$$

where

h = thickness of sand cap, ft

γ_s = submerged unit weight of sand, lb/ft^3

If no density data are available for the sand cap, a value of 1,650 kg/m^3 (103 lb/ft^3) (or 650 kg/m^3 (40.6 lb/ft^3) submerged unit weight) (Poindexter 1988) may be assumed for preliminary calculations.

Figure 1 shows the calculated factor of safety against bearing capacity failures for different dredged material strengths for an initial cap thickness of 0.3 m (1 ft), an intermediate thickness of 0.6 m (2 ft), and the final design thickness of 1 m (3.3 ft). Table 2 shows the strength that must be present in the dredged material at a factor of safety of 1.0 when the cap is on the verge of precipitating a bearing capacity failure and at 3.0, which would represent a conventional design factor of safety against a bearing capacity failure.

As an example, assume the dredged material soil strength on a current project is in the range of 479 to 958 Pa (10 to 20 lbf/ft^2). Enter Figure 1 with the strength, and move upward to intersect the diagonal line representing a particular cap thickness. Then move horizontally to read the factor of safety for bearing capacity. From this figure, it appears that the 0.3-m- (1-ft-) thick cap would be stable (FOS > 1), but the 0.6-m- (2-ft-) thick cap would be more questionable (FOS $\leq$1). The final 1-m- (3.3-ft-) thick cap would be expected to trigger a bearing capacity failure (FOS < 1). None of the cap thicknesses would produce a conventional level of protection against bearing capacity failure (e.g., FOS of 3.0).

The preceding bearing capacity calculations reveal that rapid placement of thick sand caps could engender bearing capacity failures. Such failures could result in upward displacements of dredged material and ultimately in unanticipated reductions in the actual cap thickness or exposure of the dredged material to the overlying ocean environment. These sand caps are most vulnerable to bearing capacity failures immediately after they are placed.

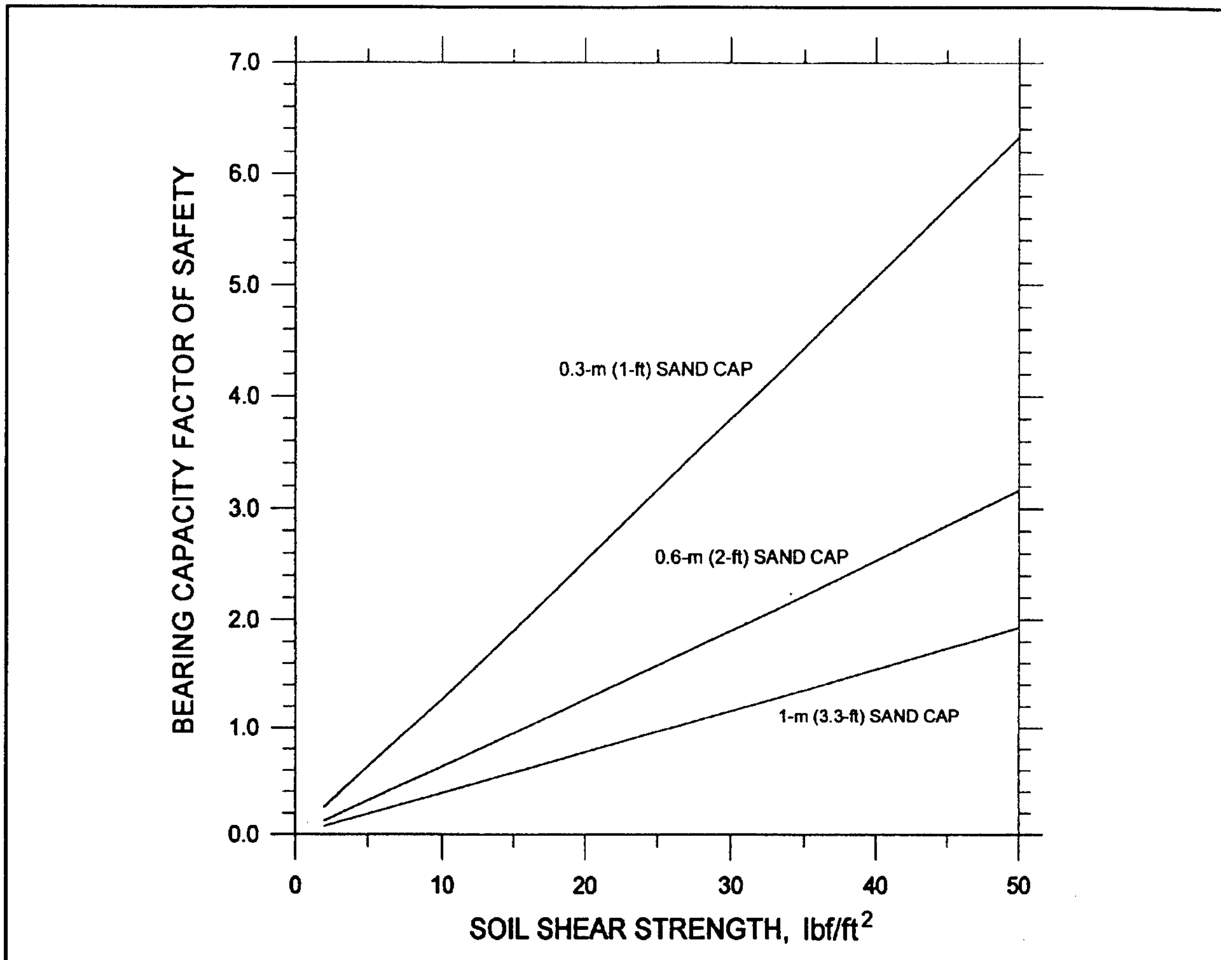

Figure 1. Bearing capacity factors of safety for different thicknesses of sand cap as a function of soil undrained shear strength (to convert soil shear strength to Pascals, multiply by 47.88)

Table 2
Soil Stregnth Required for Different Bearing Capacity Factors of Safety

Sand Cap Thickness	Soil Strength, Pa (lbf/ft^2)	
	FOS = 1.0	FOS = 3.0
0.3 m (1 ft)	383 (8)	1,149 (24)
0.6 m (2 ft)	766 (16)	2,298 (48)
1 m (3.3 ft)	1,245 (26)	3,687 (77)

If the dredged material to be capped is too soft initially to sustain a full cap thickness, it may be possible to construct a coherent cap in thinner lifts, i.e., use staged construction of the cap. After placement of a portion of the cap, the dredged material will consolidate under the load and will gain strength with time. Then the process could be repeated with additional capping material being placed and further consolidation of the dredged material allowed to occur. In this manner, it should be possible to design staged cap construction even for very soft sediments, so that the risk of bearing capacity failures can be minimized.

Infinite Slope Analysis (Slope Stability). For long slopes where the end effects on the sliding material can be neglected, an infinite slope analysis of the stability of the slope can be expressed as

$$FOS = \frac{c_u}{\gamma z (1/2 \ \sin \ 2\alpha)} \tag{5}$$

where

FOS = factor of safety against sliding

c_u = undrained shear strength, lbf/ft^2

γ = unit weight (submerged unit weight for submerged slope), lb/ft^3

z = depth at which sliding occurs, ft

α = slope angle

An infinite slope analysis is appropriate for the long, continuous slopes such as those formed in dredged material deposits. In this type analysis, the thickness of the unstable material is small compared to the height of the slope, and each vertical block of soil above the potential slip plane will have the same forces acting on it (Taylor 1954; Lambe and Whitman 1979). The infinite slope analysis will indicate the various combinations of slope angle and maximum deposit height that will result in a stable deposit. It is a simple analysis to conduct, and it is one that should be evaluated in any comprehensive slope stability analysis.

Spencer's Analysis Method (Slope Stability). A detailed analysis of potential sliding surfaces should be made using Spencer's method, which is generally considered to be the most reliable of the method of slices analysis techniques and which provides the best solutions for meeting both force and moment equilibrium requirements. Calculations can be made using the UTEXAS3 computer program developed at the University of Texas (Edris and Wright 1992).

When the program is used, piezometric levels should be assumed to coincide with mean water surface elevation. The water should be modeled with normal loads applied to the slope surface as is recommended by Edris and Wright (1992) for submerged slopes. Analysis conditions should include the dredged material prior to capping and soon after capping, if a cap will be used on the project. If stability problems are indicated in the analysis, it would be advisable to simulate consolidation for some reasonable period of time, then correlate output from a consolidation analysis (such as output from Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF)) to density and strength at that time, additional stability calculations can be made to assess the effect of consolidation on the stability of the material. In simulations with a cap, the cap should be modeled as a load normal to the slope surface.

The condition of the foundation soil (firm versus weak) beneath the soft dredged material mound has a major impact on the formation of the slope failure surface. Several failure modes should be postulated and examined individually for each project, as illustrated in Figure 2, which is based on data for a capping project in the Mud Dump Site (Rollings and Rollings 1998b). The computerized analysis using Spencer's method of slices locates the specific surface of these various potential

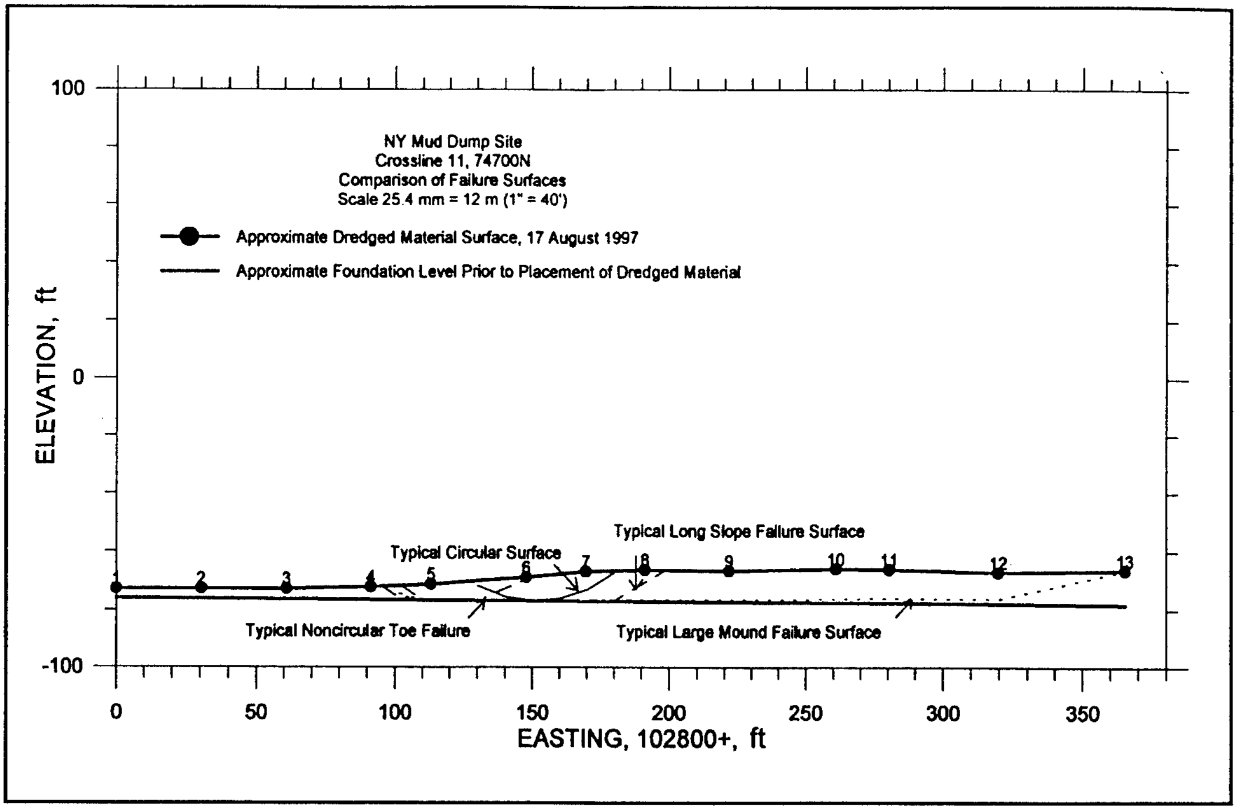

Figure 2. Example criterial failure surfaces for different modes of failure (to convert feet to meters, multiply by 0.3048)

failure modes that has the lowest factor of safety against sliding. The computer program UTEXAS3 searches for the critical surface that gives the lowest factor of safety on a grid that moves the postulated failure shape up and down the slope. By using different starting failure shapes and different grid densities, the user can examine a large range of possible failures and identify the ones with the smallest factor of safety.

The sand cap is usually assumed to be applied instantaneously and uniformly for the analysis. Realistically, this is impossible. The best stability will be achieved if the sand is placed at the toe of the slope and then worked up toward the crest of the slope. Achieving this controlled placement is problematic under actual disposal conditions, and if sand is placed toward the crest of the slope before the lower portions of the slope are covered, the factor of safety against sliding will generally be reduced.

Critical failure surfaces with the lowest factor of safety should be located for each mode of slope failure. These values of factor of safety should be compared; the failure surface with the lowest factor of safety will be the most likely to fail. Limit equilibrium analysis of this type determines only if a configuration is stable or unstable. It does not predict postsliding configuration or behavior. Once a slide occurs, the uphill scarp may be quite steep and subject to further progressive slips.

The term failure used in this technical note simply describes the results of an equilibrium analysis for bearing capacity or slope stability. The displacement, sliding, or sloughing of the slopes of a dredged material mound is not necessarily detrimental provided the material is not resuspended or left with an inadequate cap thickness. Instead, it simply represents the material adjusting itself to a configuration of static equilibrium. In this sense, the analysis needs to be kept in perspective.

Consolidation Analysis. Many soft, fine-grained dredged materials may undergo vertical strain on the order of 50 percent during the consolidation process. Therefore the objective of consolidation analysis is to determine the amount and rate of consolidation that the CAD deposit will undergo as a result of self-weight consolidation and/or surcharge loading. Because of the large strains involved in dredged materials deposits, the general Terzaghi consolidation theory is totally inadequate to model the problem. Instead a finite or large strain consolidation theory provides the proper analytical approach.

The most general and least restrictive of the many one-dimensional primary consolidation formulations was developed by Gibson, England, and Hussey (1967) and has been expanded and applied to various types of dredged material and soft soil analyses (e.g., Gibson, Schiffman, and Cargill 1981; Cargill 1985; Townsend 1987; Poindexter 1988; Townsend and McVay 1990). These models include consideration of primary consolidation with the option to include desiccation crust formation, as applicable. Secondary compression (which can be significant in these thick, soft deposits) is not explicitly considered but is implicitly included in the desiccation model. Attempts have been made to include secondary compression directly in the PSDDF model (Stark, in preparation), but this portion of the program is not currently efficacious. (It does not adequately model real dredged material behavior.) However, this model does include capability for modeling a sand or fine-grained cap and multiple dredged material layers. The most recent version of the PSDDF finite strain consolidation model, including some typical input data, is included in the desktop personal computer version of the Corps of Engineers Automated Dredging and Disposal Area Management System (ADDAMS), an assortment of dredging-related computer programs that can assist in management of dredging and disposal operations (Schroeder and Palermo 1990).

In evaluating consolidation, both the time-rate and the ultimate magnitude of consolidation should be determined separately for the contaminated sediment, the capping material, and the foundation layers, as appropriate. Then for any given time of interest, the individual settlement values for the various layers are summed to provide an estimate of the total amount of consolidation settlement to be expected at that particular time. This information can be used in conjunction with field monitoring data in the ongoing assessment of cap integrity. Any change in thickness of the capping material is of primary concern from an environmental perspective. However, the total amount of consolidation settlement, or decrease in elevation, of the cap surface over time is necessary to delineate between mound height changes caused by erosion and those accounted for by consolidation of constituent materials. Figure 3 illustrates surface elevation changes over time for various hypothetical CAD deposits with a range of initial void ratios.

CONCLUSIONS: Predictions of both consolidation behavior and shear strength/stability can be made prior to construction of a capped dredged material mound. However, the quality of the prediction will be only as good as the input data or assumptions used in lieu of site-specific data.

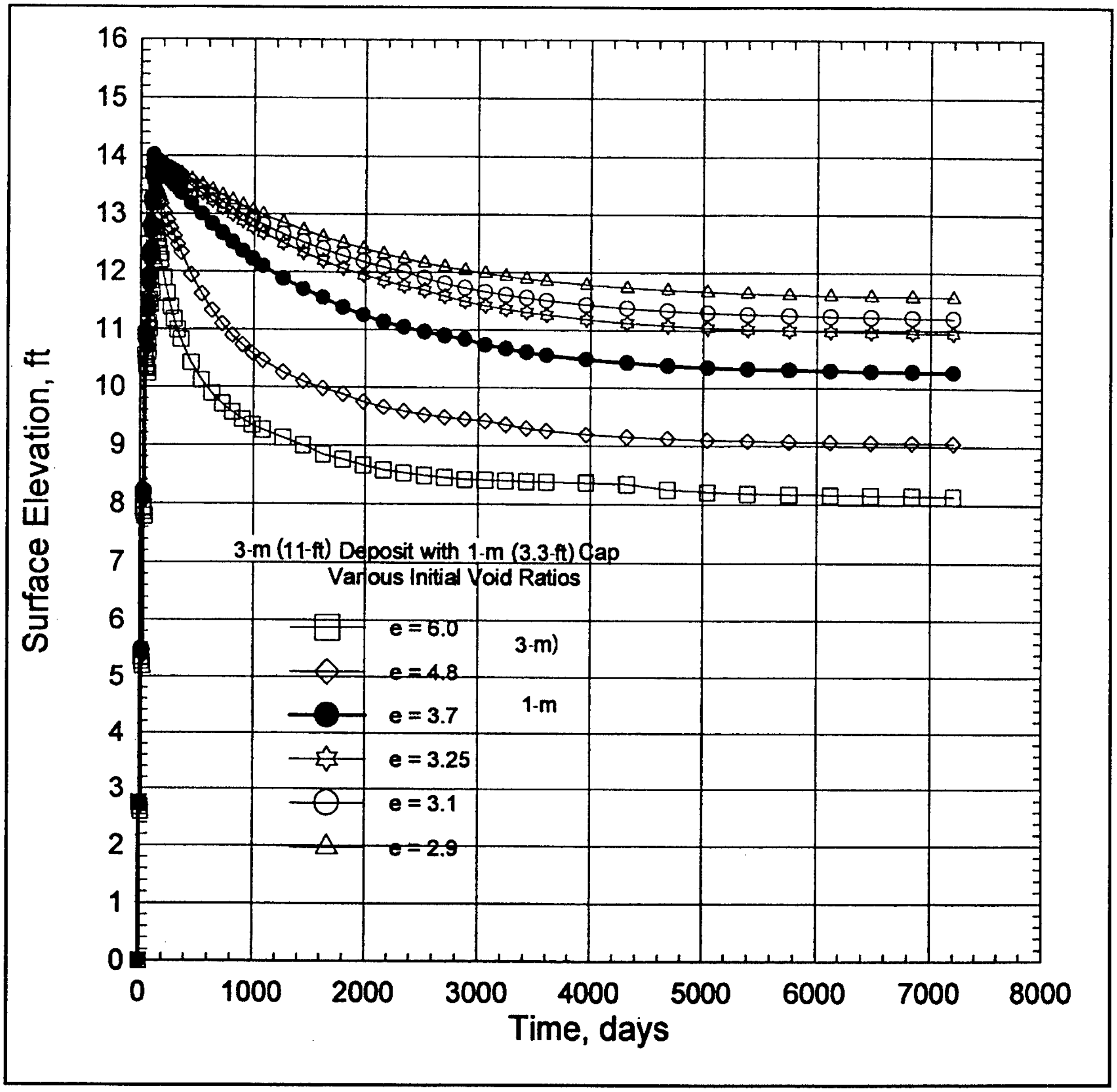

Figure 3. Example of CAD consolidation under 1-m cap with various initial void ratios in the dredged material (to convert elevations to feet, multiply by 0.3048)

Dredged material properties often vary significantly within small geographic areas. Therefore, it is critical that samples be collected and tested from any current dredging project that is to be designed.

Dredged materials are highly heterogeneous, partially because of the inherent geologic variation of natural sediments and partially because sediments from different sites may be disposed of in any specific CAD cell. Consequently, a large number of samples may be needed to adequately characterize the range of materials and conditions present at a site. Thus, it is often best to analyze the mass of data from a sampling program and to identify trends and ranges of properties. It is normally not satisfactory to try to investigate individual data points and cores.

Techniques were presented for analyzing the stability of subaqueous dredged material deposits, whether or not caps are used. Various shapes and locations of potential failure surfaces must be analyzed to determine the most critical potential failure plane.

Methods for analyzing the consolidation potential of CAD deposits were recommended. Use of finite (large) strain consolidation theory is necessary for these soft materials. Consideration must be given to potential consolidation of not only the contaminated dredged material, but also of the foundation soils and the capping material.

The behavior of dredged materials placed in contained aquatic disposal sites is a complex geotechnical phenomenon that is poorly understood. However, as more studies are undertaken and significant amounts of geotechnical data are produced, knowledge of these deposits and ability to predict their behavior both for consolidation and stability will improve.

POINTS OF CONTACT: For additional information, contact the author, Dr. Marian P. Rollings (601-634-2952, *rollinm@wes.army.mil*), or the Program Manager of the Dredging Operations and Environmental Research Program, Dr. Robert M. Engler (601-634-3624, *englerr@wes.army.mil*). This technical note should be cited as follows:

> Rollings, M. P. (2000). "Geotechnical considerations in contained aquatic disposal design," *DOER Technical Notes Collection* (ERDC TN-DOER-N5), U.S. Army Engineer Research and Development Center, Vicksburg, MS. *www.wes.army.mil/el/dots/doer/*

REFERENCES

American Society for Testing and Materials. (1999a). "Standard test method for field vane shear test in cohesive soil," Designation D 2573, *1999 Annual Book of ASTM Standards*, Volume 04.08, West Conshohocken, PA.

American Society for Testing and Materials. (1999b). "Standard test method for laboratory miniature van shear test for saturated fine-grained clayey soil," Designation D 4648, *1999 Annual Book of ASTM Standards*, Volume 04.08, West Conshohocken, PA.

American Society for Testing and Materials. (1999c). "Standard test method for one-dimensional consolidation properties of soils," Designation D 2435, *1999 Annual Book of ASTM Standards*, Volume 04.08, West Conshohocken, PA.

Cargill, K. W. (1983). "Procedures for prediction of consolidation in soft fine-grained dredged material," Technical Report D-83-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cargill, K. W. (1985). "Mathematical model of the consolidation/desiccation processes in dredged material," Technical Report D-85-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Clausner, J. E., Lillycrop, L. S., McDowell, S. E., and May, B. (1998). "Overview of the Mud Dump Capping Project design," *Proceedings, XVth World Dredging Congress*, WEDA, Las Vegas, NV.

Edris, E. V., and Wright, S. G. (1992). "UTEXAS3 slope-stability package," Instruction Report GL-87-1, 4 volumes, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

ENSR. (1997). "Summary of capping investigations," BHNIP Independent Observer Report I/O-9703, ENSR Document Number 4479-001-120 prepared for Massachusetts Coastal Zone Management Agency, Executive Office of Environmental Affairs, Boston, MA, by ENSR, Acton, MA.

Gibson, R. E., England, G. L., and Hussey, M. J. L. (1967). "The theory of one-dimensional consolidation of saturated clays; I. Finite, non-linear consolidation of thin homogeneous layers," *Geotechnique* 17, 261-273.

Gibson, R. E., Schiffman, R. L., and Cargill, K.W. (1981). "The theory of one-dimensional consolidation of saturated clays; II. Finite non-linear consolidation of thick homogeneous layers," *Canadian Geotechnical Journal*, 18.

Headquarters, U.S. Army Corps of Engineers. (1986). "Laboratory soil testing," Engineer Manual EM 1110-2-1906, Washington, D.C.

Headquarters, U.S. Army Corps of Engineers. (1987). "Confined disposal of dredged material," Engineer Manual EM 1110-2-5027, Washington, DC.

Lambe, T. W. (1951). *Soil Testing for Engineers*. John Wiley, New York.

Lambe, T. W., and Whitman, R. V. (1979). *Soil Mechanics, SI Version*. John Wiley, New York.

Ling, H. I., Leshchinsky, D., Gilbert, P. A., and Palermo, M. R. (1996). "In-situ capping of contaminated sediments: Geotechnical considerations." *Proceedings, Second International Congress on Environmental Geotechnics*, Osaka, Japan, 5-8 November 1996, International Society of Soil Mechanics and Foundation Engineering, Balkema, Rotterdam, The Netherlands, 575-580.

Mitchell, J. (1993). *Fundamentals of Soil Behavior*. 2nd edition, Wiley Interscience, New York.

Murray, P. M., Fredette, T. J., Jackson, P. E., Wolf, S. H., and Ryther, J. H. (1998). "Monitoring results from the Boston Harbor Navigation Improvement Project Confined Aquatic Disposal Cell." *Proceedings, XVth World Dredging Congress*, WEDA, Las Vegas, NV, June 28-July 2, 1998. 415-430.

Palermo, M. R., Maynord, S., Miller, J., and Reible, D. (1998a). "Guidance for in-situ subaqueous capping of contaminated sediments," EPA 905-B96-004, U.S. Environmental Protection Agency, Great Lakes National Program Office, Chicago, IL.

Palermo, M. R., Clausner, J. E., Rollings, M. P., Williams, G., Myers, T., Fredette, T. J., and Randall, R. E. (1998b). "Guidance for subaqueous dredged material capping," Technical Report DOER-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS. *www.wes.army.mil/el/dots/doer/*

Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974). *Foundation engineering*. 2nd edition, John Wiley, New York.

Poindexter, M. E. (1987). "Consolidation properties of dredged material." *Proceedings, Consolidation Prediction Symposium*. Florida Institute of Phosphate Research and University of Florida, Lakeland, FL.

Poindexter, M. E. (1988). "Behavior of subaqueous sediment mounds: Effect on dredged material disposal site capacity," Ph.D. Diss., Texas A&M University, College Station, TX.

Prandtl, L. (1921). "Über die eindringungsfestigkeit (härte) plastischer baaustoffe und die festigkeit von schneiden' (on the penetrating strengths (hardness) of plastic construction materials and the strength of cutting edges), *Zeit. angew. Math. Mech.* 1(1).

Rollings, M. P. (1994). "Geotechnical considerations in dredged material management." *Proceedings, First International Congress on Environmental Geotechnics*, Edmonton, Alberta, Canada, 10-15 July 1994. International Society of Soil Mechanics and Foundation Engineering and Canadian Geotechnical Society, BiTech Publishers, Richmond, British Columbia, Canada, 21-32.

Rollings, M. P., and Rollings, R. S. (1996). *Geotechnical materials in construction*. McGraw-Hill, New York.

Rollings, M. P., and Rollings, R. S. (1998a). "Consolidation and related geotechnical issues at the 1997 New York Mud Dump Site." *Proceedings, XVth World Dredging Congress*, WEDA, Las Vegas, NV, June 28-July 2, 1998. 1-16.

Rollings, R.S., and Rollings, M.P. (1998b). "Shear failures in the New York Harbor Disposal Mound and implications for design of sand caps." *Proceedings, XVth World Dredging Congress*, WEDA, Las Vegas, NV, June 28-July 2, 1998.

Schroeder, P. R., and Palermo, M. R. (1990). "The Automated Dredging and Disposal Alternatives Management System (ADDAMS)," EEDP Technical Note 06-12, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS. *www.wes.army.mil/el/dots/eedptn.html*

Sowers, G. B., and Sowers, G. F. (1970). *Introductory soil mechanics and foundations*. MacMillan, New York.

Stark, T. "Program documentation and user's guide: PSDDF Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill" (in preparation), U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Taylor, D. W. (1954). *Fundamentals of soil mechanics.* John Wiley, New York.

Teng, W. C. (1962). *Foundation design.* Prentice-Hall, Englewood Cliffs, NJ.

Terzaghi, K., and Peck, R. B. (1976). *Soil Mechanics in Engineering Practice.* Wiley Interscience, New York.

Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil mechanics in engineering practice.* 3rd edition, Wiley Interscience, New York.

Townsend, F. C. (1987). "Clay waste pond reclamation by sand/clay mix or capping," *ASCE Journal of Geotechnical Engineering* 115(11), 1647-1666.

Townsend, F. C., and McVay, M. C. (1990). "State-of-the-art: Large strain consolidation predictions," *ASCE Journal of Geotechnical Engineering* 116(2), 222-243.

U.S. Department of Defense. (1964). "Military Standard: Subgrade, subbase, and test method for pavement base-course materials," MIL-STD-621A, Washington, DC.

U.S. Department of the Army. (1987). "Field Manual FM 5-530: Materials testing," Department of the Army FM 5-530, Department of the Navy NAVFAC MO-330, and Department of the Air Force AFM 89-3, Washington, DC.

U.S. Environmental Protection Agency and U.S. Army Corps of Engineers. (1991). "Evaluation of dredged material proposed for ocean disposal (Testing manual)" (Green Book), EPA/503/8-91.001, U.S. Environmental Protection Agency, Office of Water, and Department of the Army, U.S. Army Corps of Engineers, Washington, DC. *www.epa.gov/OWOW/oceans/gbook*